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FOUNDATIONS FOR NARROW BASED TRANSMISSION TOWERS

Report No 87-18-K

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ABSTRACT

Single pier V2S narrow based transmission tower foundations are required to resist a shear force and overturning moment as a result of lateral loading of the transmission tower structure. This report presents the results obtained using the PADLL program to select pier size for the design loads and performance criteria for footings in Leda clay and in compact sand. Comparative analyses are offered for pier response resulting from the addition of a 4.0 m square by 2.0 m deep concrete cap to the pier and to replacement of the top 2.0 m of soil with well compacted fill. The group influence effect on four footing foundations is discussed by means of a literature review. Areas requiring further study are indicated.

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TABLE OF CONTENTS

	Page
INTRODUCTION	1
DESIGN CONCEPTS	1
DESIGN CONSIDERATIONS FOR SINGLE PIER FOUNDATIONS	2
Physical Dimensions	2
Attachment of Tower Structure to the Foundation	2
Frost Heave Associated with the Cap	2
Lateral Deflection and Rotation	3
EXAMPLES OF SINGLE PIERS	3
CONSTRUCTABILITY AND CONSTRUCTION EFFECTS ON FOUNDATION STRENGTH	4
GROUP EFFECT IN COHESIVE AND COHESIONLESS SOILS	5
TEST RESULTS	6
Group Effect on Uplift Capacity of Drilled Piers in Sand	6
Group Effect on Uplift Capacity of Piles in Clay	6
SUMMARY AND CONCLUSIONS	7
AREAS REMAINING TO BE INVESTIGATED	7
REFERENCES	9
TABLES I to IV	10 - 14
FIGURES 1 to 8	15 - 22
APPENDIX A	23
DISTRIBUTION	last page

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FOUNDATIONS FOR NARROW BASED TRANSMISSION TOWERS

INTRODUCTION

Narrow based towers are being considered for the proposed new transmission lines to increase the efficiency of land use in urban centres and in agricultural fields.

Single steel poles have been used in areas where the improved appearance is needed in addition to the efficient use of right-of-way. The cost of these steel poles is prohibitive for their use in all areas. Therefore a new design of narrow based lattice towers is being considered. Typically, the base of these towers is about 3 m square. The foundations for this type of narrow based tower are required to resist large loads caused by wind and ice on conductors and structure. Two possible foundation schemes for the narrow based towers are (i) four footings, one for each tower leg and (ii) one single foundation for the tower.

This report discusses the design considerations of the above two foundation schemes with the help of example cases. Areas of further study are identified.

DESIGN CONCEPTS

The single footing must resist the lateral load and overturning moments from the tower and must also accommodate the base of the tower which is approximately 3.3 m square in size. In addition, there will be some performance criteria on footing movement that must be satisfied. Several possibilities for this type of foundation are shown in Figure 1.

In the case of four independent footings, the loading from the narrow based tower would typically put two of the four footings in uplift and two footings in compression. The force applied to each footing could change or reverse with varying loading combinations. One of the advantages of the four footing tower foundations is that movement of the footings as a result of the applied loads will cause the load to be redistributed by the tower structure to the remaining footings.

The other types of foundations include a thick reinforced concrete slab anchored to soil. Two methods that could be used to anchor the concrete slab to the soil are driven steel piles and rock or soil anchors. An

inclination to the piles or anchors may reduce the amount of overlap between the shear zones surrounding the anchors.

DESIGN CONSIDERATIONS FOR SINGLE PIER FOUNDATIONS

Physical Dimensions

The physical dimensions of a drilled pier foundation required for the anticipated design loads and performance criteria can be determined using the Fortran program PADLL (Pier Analysis and Design for Lateral Loads) /1/. PADLL incorporates the four spring model in combination with a finite element beam model for the pier.

Analyses of footings using the PADLL program have shown that the lateral load and moment capacity are increased while groundline deflections and rotations are decreased as the total embedment length is increased for any given pier diameter. Unfortunately there are limitations to the depth that can be drilled with the currently used equipment. Similar limitations apply to the maximum diameter of the augers. The design dimensions for a drilled pier depend on the soil conditions and the depth of the water table. The maximum depth would be approximately 13.5m and the maximum diameter would be 2.5 m with current construction equipment.

The cap will be constructed in a square shape, with sides of approximately 4.0 m. In the analysis of the foundation using PADLL, the cap dimension was entered as 4.0 m diameter because of a program limitation requiring all piers to be round.

Attachment of Tower Structure to the Foundation

The width of the tower base is approximately 3.3 m, while the diameter of the pier typically may be 2.4 m in diameter. A reinforced concrete cap will be required to transfer moments and shears from the tower to the foundation pier. Anchor bolts embedded in the concrete will be used for attaching the tower to the cap.

Frost Heave Associated with the Cap

In frost susceptible soils, the footing may be subjected to tremendous uplift forces due to adfreeze and basal heave effects. Adfreeze stresses have been measured by Penner and Goodrich /2/ for 323 mm steel piles embedded in Leda clay. The maximum adfreeze stress on the sides of the pile was 105.5 kPa occurring in December. The average adfreeze stress for the winter months shown in Table 1 (D) is 76.5 kPa. The work of Penner and Goodrich also shows that the adfreeze stresses decrease for increasing diameter of pier. This average value of adfreeze stress agrees closely with the average value of approximately 80 kPa (Figure 2) that was obtained by Penner /3/ during an earlier study using steel piles of similar size. Adfreeze stresses of 25kPa on a concrete wall were reported in this paper as shown in Figure 3.

Basal frost heave pressures measured by Penner/3/ for steel plates were in the range of 1000 - 1800 kPa as shown in Figure 4. Appendix A shows the frost heave calculations for a 2.4 m diameter pier with an embedment length

of 12.5 m and a 4.0 m square cap of 1.5 m depth, (Figure 5) in a region with a 2.0 m frost penetration depth. The results are summarized below:

Uplift capacity of unfrozen length of pier = 3885 kN
Uplift due to adfreeze on sides of 4.0m cap = 1836 kN
Uplift due to basal heave under the cap = 17214 kN

Therefore, the bottom of the cap should be well below the frost penetration depth to prevent basal heave.

Lateral Deflection and Rotation

The performance criteria that is currently specified for the single pole footing, places a limit on groundline deflection of approximately 20 mm. When specifying a performance criteria for a single pier foundation, limits for both groundline deflection and rotation are both important. Groundline rotation of the pier would result in some tilting of the tower structure and therefore the amount of allowable groundline rotation will be established on an aesthetical basis. Maximum groundline deflections are also limited to keep the soil stresses within the yield limit so that large permanent deflections do not occur.

It is important to consider both total and non-recoverable deflections/rotations when specifying performance criteria for single pier foundations. Figure 6 shows a typical moment-deflection graph. The loading of the pier results in a certain amount of permanent deflection, ie, if a pier loaded to a certain level is unloaded, there is an amount of deflection that is non-recoverable after unloading. Further loading of this pier, to the previously attained level, will result in a slightlygreater amount of total deflection than previously measured. The magnitude of non-recoverable deflection is greater for higher levels of loading. Effects of repeated and reversed loading on the total deflection should also be considered.

EXAMPLES OF SINGLE PIERS

In this section, two test cases of single pier foundations for a V2S narrow based tower in Leda clay and sand are considered. Table II shows the design loads for V2S type towers. The loads for the low towers were used for pier design in Leda clay and loads for the high tower were used for the sand site. Table III shows the soil parameters used in the foundation design. The pier diameter and alternative foundation shapes considered for this comparative analysis are summarized in Table IV.

The PADLL program was used to estimate the maximum groundline deflections and rotation under the design loads for piers with and without an enlarged cap. An enlarged cap of 4m diameter and 2m depth was used for comparison purposes. The embedment depth of a uniform diameter pier of 2.4 m was calculated by the PADLL program to limit the maximum groundline deflection to 20 mm for each of the soil conditions considered. The effect of replacing the top 2 m of Leda clay with a well compacted granular fill was also examined.

Table V summarizes the results of PADLL analyses. The following inferences can be drawn from this summary table:

1. The footing with the enlarged cap of 4.0 m diameter by 2.0 m thickness shows an improvement in ultimate lateral load capacity of 12% as compared to the uniform 2.4 m diameter pier. Improvements of up to 25% in the ultimate lateral load capacity are realized for footings with an enlarged cap when the top 2.0 m of Leda clay is replaced by well compacted class "A" granular fill.

2. Reductions in groundline deflection of 32% were achieved for footings with an enlarged cap compared to the uniform 2.4 m diameter footing. The amount of reduction increased to 68% when the top 2.0 m of Leda clay was replaced by well compacted granular "A" fill.

3. Reductions in total rotation of 29% were achieved for footings with an enlarged cap compared to the uniform 2.4 m diameter footing. The amount of reduction increased to 63% the top 2.0 m of Leda clay was replaced by well compacted granular "A" fill.

4. Piers founded in sand have been analyzed for a uniform diameter of 2.4 m only. Attempts to analyze footings with caps were unsuccessful because the depth to diameter limitation would have been less than the minimum required by PADLL for analysis. Foundations in sand can carry higher loads with reduced embedment lengths than can footings in Leda clay.

5. The amount of non-recoverable deflections and rotations were compared for two uniform diameter piers of 2.4 m diameter x 13 m embedment length, one founded in a soil with a stiff upper layer on a soft clay deposit, and the other with a uniform deposit of soft clay. The results show a greater amount of non-recoverable deflection and rotation for the soil profile with the stiff upper layer. This is contradictory to what would be observed when a plate load test is conducted on two soils with different levels of preconsolidation pressure in that the greatest non-recoverable deformation would be expected for the soil with the lowest preconsolidation pressure. PADLL does not require a value for the unloading modulus. It is assumed that it is determined from the pressuremeter modulus of deformation, the result of which does not give a reasonable estimate of non-recoverable deflection. Thus it should not be used for design in its present form.

6. Improvement to the surface layer of soil to depths as previously described does not affect the ultimate capacity of a pier of a given size but does allow the embedment length to be reduced while maintaining less than 20 mm groundline deflection.

CONSTRUCTABILITY AND CONSTRUCTION EFFECTS ON FOUNDATION STRENGTH

In clay, the possible reduction in foundation strength for drilled piers is related to changes in the adhesion between the pile and the soil. "The adhesion has been found to be less than the undrained cohesion before installation, mainly because of the softening of the clay immediately adjacent to the footing surface. This softening may arise from three causes:

(a) adsorption of moisture from the wet concrete;

(b) migration of the water from the body of the clay toward the less highly-stressed zone around the borehole;
(c) water poured into the boring to facilitate operation of the cutting tool"/4/.

The softening due to factors (c) and (b) can be reduced by good drilling technique and speedy concreting operations. Some softening due to (a) and (b) is unavoidable.

Piers drilled into sand will require casing or drilling fluid to support the walls during the sinking of the hole to depths below the water table. Subsequent removal of the casing during the concreting may disturb the wall surface to some extent.

For both clay and sand, the construction sequence for drilling the four holes for the footing will likely have to be such that one of the holes is drilled, concreted and sufficient time allotted for the concrete to set prior to drilling of a subsequent hole for another footing in the group. These measures are to avoid disturbance of the fresh concrete.

GROUP EFFECT IN COHESIVE AND COHESIONLESS SOILS

As previously stated the bearing capacity of a footing is significantly greater than its uplift capacity. The discussion of a group effect for drilled piers will be limited to the uplift capacity.

The ultimate uplift resistance of a single pile in clay is:

$$Q_u = \pi B\ Ca\ D + W \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Where B= Pile diameter
D= Embedment length
Ca= Adhesion
W= Weight of the pile

The ultimate uplift resistance of a single pile in sand is :

$$Q_u = 1/2\ \pi\ B\ D^2\ \gamma Ku \tan \emptyset + W \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Where B= Pile diameter
D= Embedment length
γ = Unit weight of soil
Ku= Empirical uplift coefficient
$\emptyset$ = Angle of internal friction of soil

The value for Ku back-calculated from field tests with footings spaced far apart has been related to the depth to diameter ratio and to the friction angle of the soil.

The variation of the adhesion factor with undrained cohesion was determined by Tomlinson (1957) for concrete piles embedded in clay and subjected to downward loads and is shown as Figure 7. Das, Azim and Echelberger (1986) have compared the adhesion factors from a number of field uplift tests by other authors on cast in-situ bored concrete piles in clay and found

reasonable agreement between the adhesion factor calculated for the push in and pull out tests on piles.

The close proximity of the footings in a four footing foundation has raised concern amongst designers that the shear zones around a pier in uplift may reduce the uplift capacity due to the overlapping of the shear zones. The shape of the shear zone surrounding the pier at failure in uplift has yet to be agreed on in the literature. The capacity of a group of piles with overlapping shear zones will be less than that of an isolated pier because the soil that is within the shear zone of one pile will not be able to offer any resistance to the uplift of adjacent piles in the group.

The uplift capacity of a group of drilled piers with a given spacing could be predicted as: (a) less than the uplift capacity of a single pier multiplied by the number of piers in the group; or (b) greater than the uplift capacity of an "equivalent footing" with a diameter calculated from the perimeter of the footing group, provided that the calculated capacity is less than that calculated in (a).

The uplift capacity of the "equivalent footing" in sand is calculated using the Ku value determined for the equivalent diameter in the D/B ratio. In clay, one should use the Ca value for the portion of the "equivalent perimeter" that has clay to concrete contact and use the Cu value for the portion with clay to clay contact.

TEST RESULTS

Group Effect on Uplift Capacity of Drilled Piers in Sand

This section summarizes the results of a field testing program/7/ in which footings of 0.71 m diameter and 3.66 m embedment length and spacing of 1 and 1.5 diameters (edge to edge) were simultaneously pulled.

The analysis of the test data showed a reduction in the uplift coefficient Ku value of 30 to 35 per cent from the single footing value obtained using the equation suggested by Meyerhof and Adams (1968) due to the overlapping zones of influence for the two footings.

The load deflection behaviour of footings in a group may be different from that of single footings. "Most of the test footing [pairs] failed suddenly with deflections of only 5 mm to 6 mm, most of which occurred after the applied load reached 65% of the ultimate load. This is less than the deflection at failure (12 mm to 25 mm) recorded in earlier tests on single footings (Cragg, 1985)"/7/.

Group Effect on Uplift Capacity of Piles in Clay

The group effect has been studied by Das et al (1986) using aluminum piles with circular cross section and diameters of 25.4 mm and 50.8 mm. Based on the results of their uplift tests on group piles of the configurations shown in Figure 8 in soft clay (Cu = 9.97 KN/m^2), the following conclusions can be drawn:

1. For a given group configuration and embedment ratio, the group efficiency increases almost linerarly with the centre to centre spacing to depth, S/D ratio to reach 100%.

2. Pile group efficiencies as a function of pile spacing, plotted for piles with depth to diameter ratios of 12, show that irrespective of the number of piles in a group, an efficiency of about 100% is reached at S/D ratios between 5.5 and 7 with an average of approximately 6. Similarly for piles with depth to diameter ratio of 15, efficiency of 100% is reached at an S/D ratio of about 7.5. The results of Meyerhof and Adams (1968) when projected upwards show group efficiencies of 100% at S/D = 9.2. So, in general, it appears that for group efficiencies of 100%

$$(S/D) \simeq 0.5\ (L/D) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

3. For similar S/D ratios (< about 0.5 (L/D)) and L/D ratios, the group efficiency decreases with the number of piles in the group.

4. For similar values of S/D and number of piles in a group, the magnitude of efficiency appears to decrease with the increase of embedment ratio (L/D).

SUMMARY AND CONCLUSIONS

Single pier footings apply a different set of loading conditions that the soil must resist than do the four footing tower foundations. Performance criteria for single pier narrow based tower foundations need to be modified to set allowable limits on footing rotation because the rotation of the footing will be visible as a tilting of the tower structure.

A cap is necessary to attach the narrow base tower structure to the single pier foundation. It is imperative that the depth of frost penetration remain well above the base of the cap because of the tremendous basal frost heave stresses that can be produced.

Improvements to the soil properties of the first 2 m around a footing was not found to increase the ultimate capacity of the footing but a small decrease in the amount of groundline deflection was noted.

Piers with the 4.0 m diameter by 2.0 m thick concrete cap experienced an increase in the ultimate lateral load capacity and a decrease in the total groundline deflection and a decrease in the total rotation as compared to a uniform 2.4m diameter pier of the same length.

The uplift capacity of a footing group is a function of the depth to diameter ratio and the spacing to diameter ratio.

AREAS REMAINING TO BE INVESTIGATED

Cyclic loading of single pier foundations needs to be investigated because of the unknown accumulative effect this may have on total and non-recoverable deflections and rotations. Field measurements of single pier footings subject to overturning moment have shown differing values of the loading and unloading modulus. PADLL does not require both modulii values

and does not give reliable predictions of non-recoverable deflection and rotation.

A sensitivity analysis should be conducted for single pier footings to assess the influence of the following parameters: pier diameter, cap dimensions, embedment length and soil properties. Such analyses would be useful in optimizing the footing dimensions. A set of cost-benefit analyses could then be conducted based on material, labour and equipment development costs. This may result in a modification to the footing dimensions.

Investigations need to be undertaken to determine if the two footings in compression have an effect on the uplift capacity of the remaining two footings in the four footing narrow based tower configuration.

The effect of repeated and reversed loading on four footing narrow based tower foundations needs to be investigated.

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TABLE I

Calculated Adfreeze Stresses* (kPa) on Steel Pipe Piles (after Penner/2/)

Thompson, Manitoba

(A) 1972/73, FI 2599°C-days, Maximum stress before release--surface gravel layer adfrozen to piles

S6 (dia. 169 mm) - 380**
S12 (dia. 323 mm) - 179**
S18 (dia. 458 mm) - 124**

(B) 1973/74, FI 3189°C-days, Average of 3 piles for each diameter--gravel layer isolated from pile

	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May
S6	35.9	38.9	79.9	59.1	36.1	19.8	13.4
S12	56.9	29.7	47.0	35.1	23.2	14.1	12.0
S18	37.1	22.6	46.2	37.8	29.1	17.6	6.2

Winter	
Average	Maximum
40.5	117
31.1	76
28.1	69

(C) 1974/75 FI 2406°C-days, Average of 3 piles for each diameter--gravel layer isolated from pile

	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May
S6	32.4	39.2	47.8	39.7	26.1	20.6	**
S12	18.5	20.2	24.2	21.4	12.1	10.4	**
S18	10.3	18.7	29.2	24.1	16.0	12.4	**

Winter	
Average	Maximum
34.3	69
17.8	55
18.5	48

Ottawa, Ontario

(D) 1971/72 FI, 1067°C-days, Average of 3 piles of each diameter--no gravel layer

	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May
S6	***	137.0	75.8	88.2	73.1	***	***
S12	***	105.5	64.8	62.7	72.4	***	***

Winter
Average
93.8
76.5

* Adfreeze stress = measured force/surface area of pile in frozen ground.
** Maximum reached in the Dec.-Jan. period.
*** No frost in the ground.

TABLE II

Foundation Loads for V2S Narrow Based Towers

	Shear (kN)	Moment (kN-m)
High Tower	245	18,984
Medium Tower	226	14,916
Low Tower	222	12,760

TABLE III

Soil Properties

Case	Depth to Bottom of Layer (m)	Total Unit Weight (kg/cu.m)	Modulus of Deformation (MPa)	Friction Angle (deg)	Cohesion (kPa)
1. Leda Clay	25	1,600	12	0	31
2. Leda Clay with Sand Fill	2	2,000*	50*	40	0
	25	1,600	12	0	31
3. Leda Clay with Compacted Class 'A' Fill	2	2,200**	99**	40	0
	25	1,600	12	0	31
4. Compact to Dense Sand	2	1,680	7	36	0
	4	1,680	21	36	0
	6	1,680	35	36	0
	8	1,680	49	36	0
	10	1,680	63	36	0
5. Compact to Dense Sand with Compacted Fill on Top	2	2,000*	50*	40	0
	4	1.680	21	36	0
	6	1,680	35	36	0
	8	1,680	63	36	0
	10	1,680	63	36	0

* Class 'B' fill
**Well compacted Class 'A' granular backfill

TABLE IV

Foundation Size Required to Meet Groundline Deflection Criteria

Case	Cap Diameter (m)	Cap Depth (m)	Pier Diameter (m)	Total Embedment (m)
(1) no cap			2.4	13
(1) cap	4	2	2.4	13
(2) no cap			2.4	12.8
(2) cap	4	2	2.4	12.8
(3) no cap			2.4	13.2
(3) cap	4	2	2.4	12.3
(3) no cap			4.0	9.0
(3) no cap			2.4	13.0
(4) no cap			2.4	8.7
(5) no cap			2.4	7.8

TABLE V

Pier Response as Predicted by PADLL

Soil	Pier Embedment and description (2.4 m diameter)	Design Lateral Load (kN)	Design Moment (kN.m)	Ultimate Capacity Lateral Load (kN.m)	Ultimate Capacity Moment (kN.m)	Ultimate Capacity Increase with cap	Total Groundline Deflection (mm)	Percentage Reduction with cap	Non-recoverable Groundline Deflection (mm)	Total Rotation (m Rad)	Percentage Reduction with cap	Non-recoverable Rotation (m Rad)
Leda clay soft uniform	13.0 m with no cap	222	12,760	288	16,553		18.4		10.1	2.52		1.38
	13.0 m with cap	222	12,760	323	18,572	12%	12.6	32%	6.06	1.79	29%	0.86
Leda clay with top 2 m replaced by well compacted Class 'B' material	12.8 m with no cap	222	12,760	283	16,279		18.5		12.9	2.56		1.79
	12.8 m with cap	222	12,760	325	18,708	15%	9.41	49%	5.95	1.44	44%	0.91
Leda clay with top 2 m replaced by well compacted Class 'A' fill	12.3 m with no cap	222	12,760	275	15,796		18.8		14.6	2.74		2.14
	12.3 m with cap	222	12,760	344	19,791	25%	6.07	68%	3.97	1.02	63%	0.67
	4.0 m diameter x 9.0 depth - no cap	222	12,760	264	15,195		18.7		15.5	3.75		3.12
	13.0 m with no cap	222	12,760	290	16,669		18.2		14.6	2.47		1.99
Compact sand	8.7 m with no cap	245	14,984	454	27,813		18.2		8.7	3.00		1.45
Compact sand with top 2 m replaced with well compacted Class 'B' fill	7.8 m with no cap	245	14.984	404	24,726		18.4		11.3	3.56		2.18

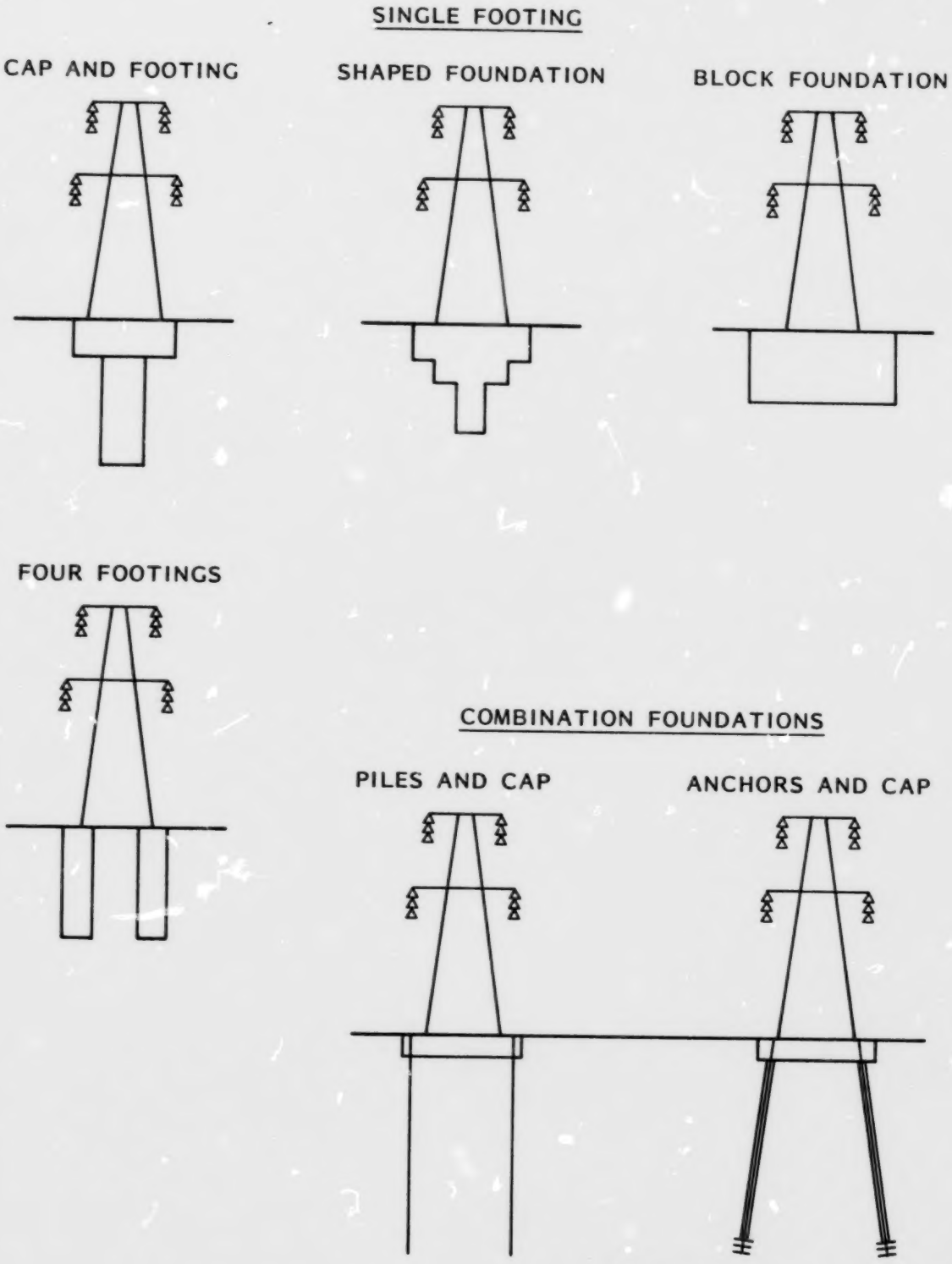


FIGURE 1

FOUNDATION POSSIBILITIES FOR NARROW BASED TRANSMISSION TOWERS

173682-RD

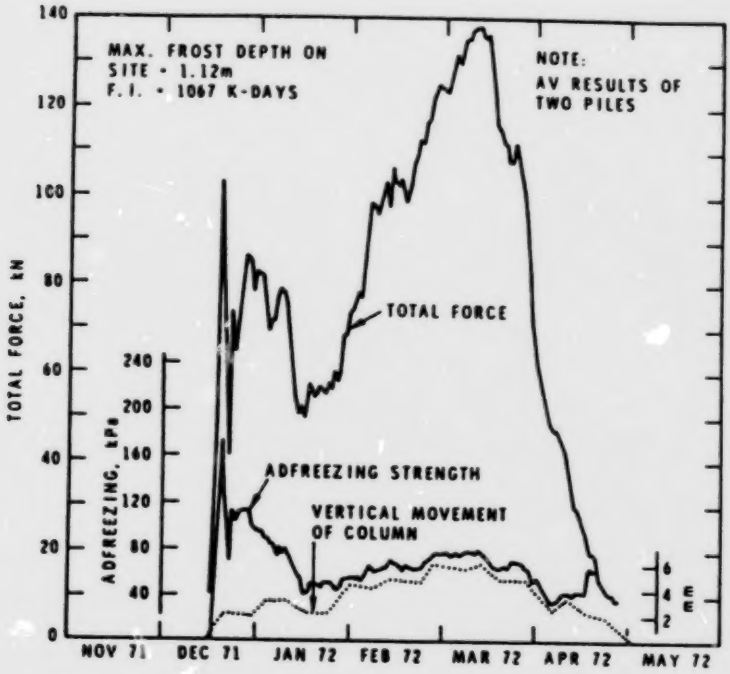


FIG. 2

Total force and adfreeze strength for 323.9-mm diameter steel pile, 1971-1972
(after Penner/3/)

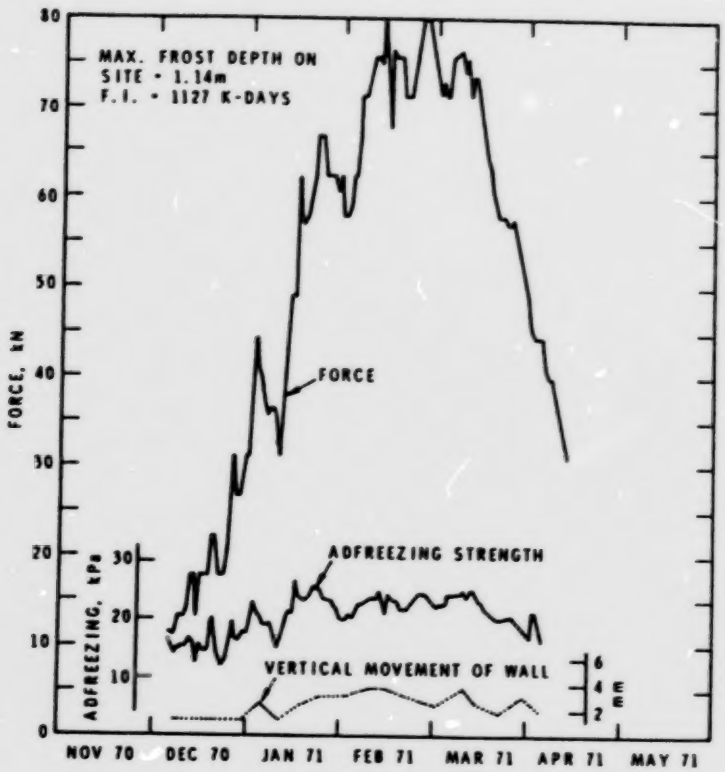


FIG. 3

Force measurements, calculated adfreeze strengths, and vertical movements of concrete block wall during heaving period, 1970-1971

(after Penner/3/)

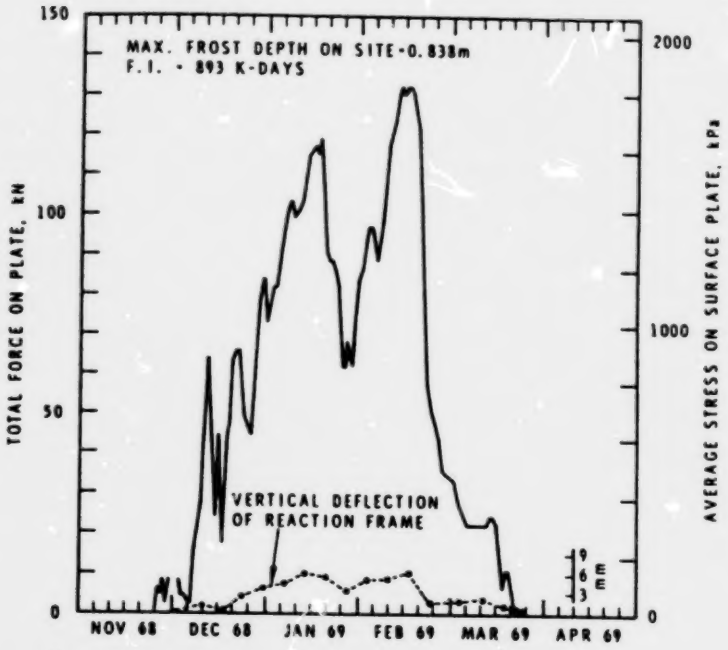


FIG. 4

Force measurements and pressures on 0.305-m diameter circular footing and vertical movement of footing, 1968-1969
(after Penner/3/)

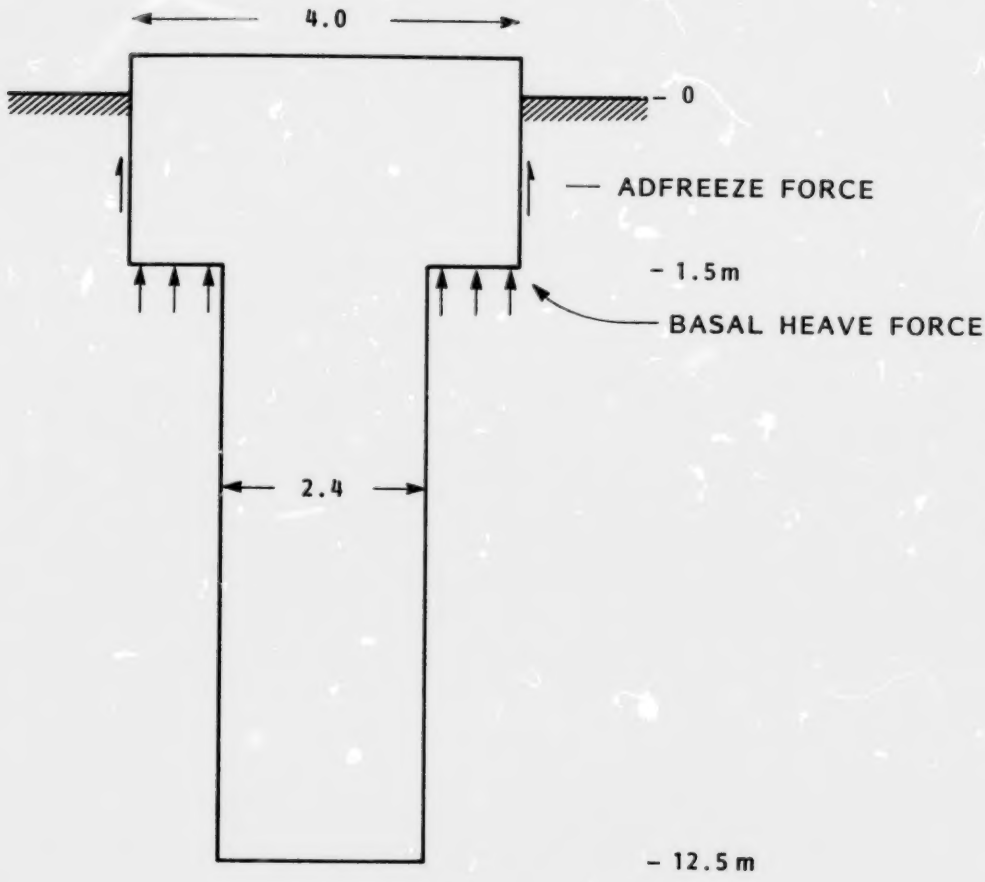


FIGURE 5
FROST HEAVE FORCES ON SINGLE PIER FOUNDATION

173686-RD

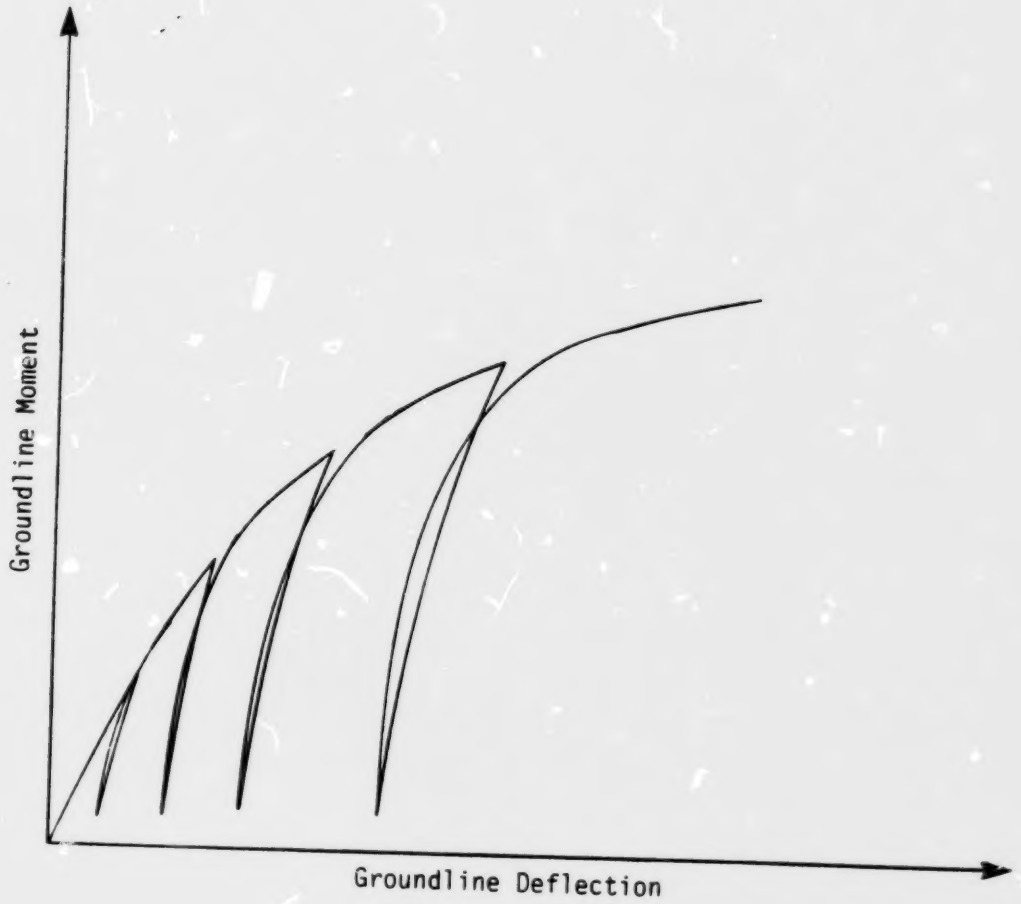


FIGURE 6

TYPICAL LOAD DEFLECTION CURVE
FOR A SINGLE PIER FOOTING

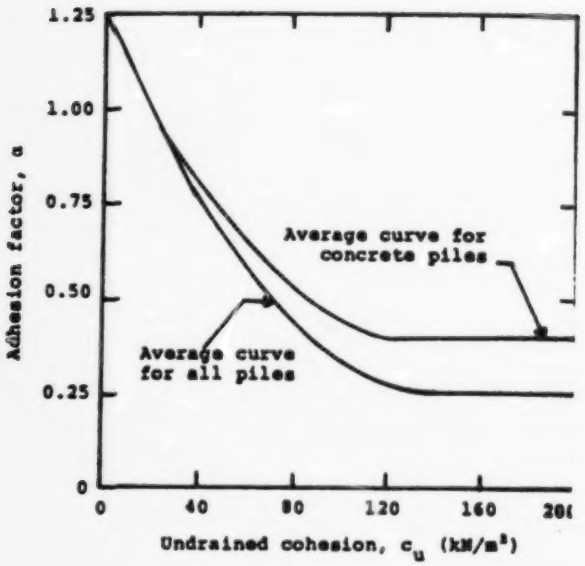
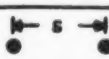
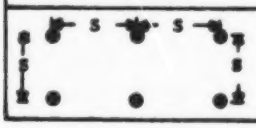
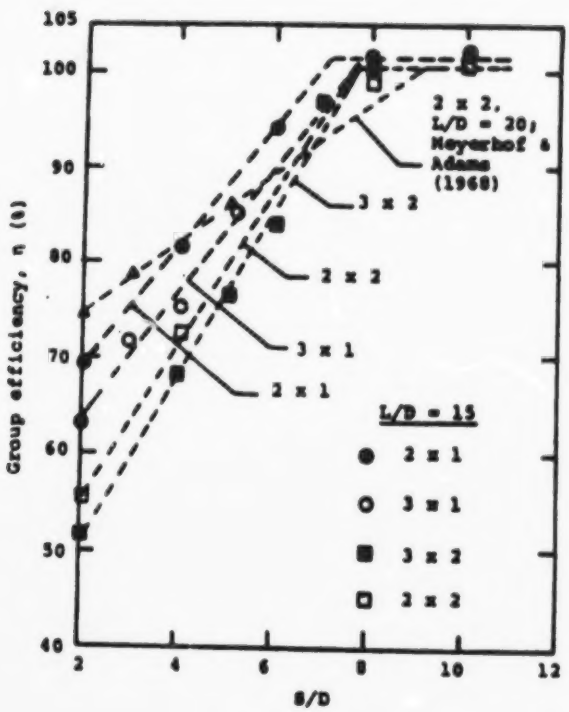


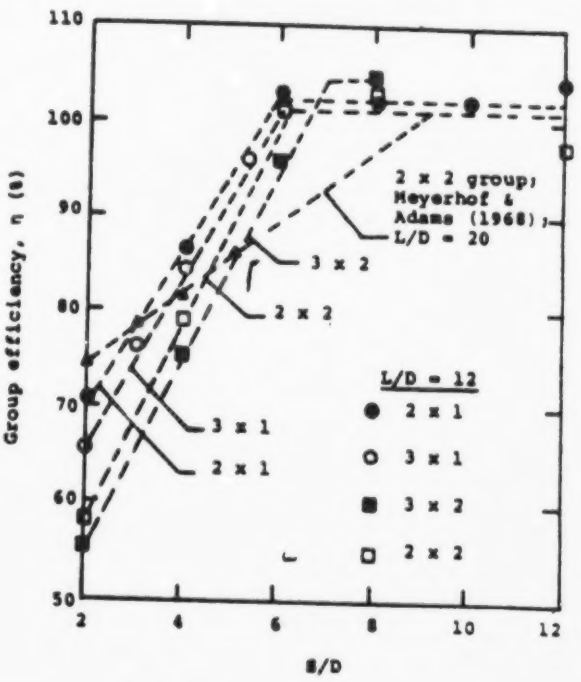
Fig. 7 Tomlinson's (1957) Recommended Variation of Adhesion Factor for Push-in Tests on Piles in Clay (after Das/8/)

Pile configuration	Designation
	2 x 1
	3 x 1
	2 x 2
	3 x 2

Configuration of Group Piles Tested



Variation of Experimental Group Efficiency With Center-to-Center Pile Spacing (L/D = 15)



Variation of Experimental Group Efficiency With Center-to-Center Pile Spacing (L/D = 12)

FIGURE 8

GROUP INFLUENCE EFFECTS ON MODEL PILES IN UPLIFT (AFTER DAS/8/)

APPENDIX A

Calculation of Uplift Forces

Concrete cap is 4.0 m square x 1.5 m deep

Frost depth is 2.0 m

Total embedment length of pier is 12.5 m

Uplift due to adfreeze on sides = 4 x 4 x 1.5 x 76.5
= 1836 kN

Uplift due to basal heave = $(4 \times 4 - \frac{\pi(2.4)^2}{4}) \times 1500$

= 17,214 kN

Therefore Total Uplift = 19,050 kN

Uplift Capacity of Pier

Embedment length available to resist the frost heave uplift is 10.5 m.

Pier is in clay (C_u = 30 kPa)

$Q_u = \pi B D \quad C_u + W$

$= \pi \times 2.4 \times 11 \times 0.6 \times 30 + \pi \times 2.4 \times 11 \times 23 + 4 \times 4 \times 1.5 \times 23$

= 3,885 kN

The uplift capacity is less than the uplift from the frost heave due to adfreeze and basal heave. The top cap must extend well below the frost penetration depth to prevent the basal heave.

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